Chapter 4
Special Features and Considerations

4-1. Sediment Control Structures

a. General. Two basic types of control structures are used:

(1) stabilizers designed to limit channel degradation and

(2) drop structures designed to reduce channel slopes to effect nonscouring velocities.

These structures also correct undesirable, low-water-channel meandering. Gildea (1963) has discussed channel stabilization practice in USAED, Los Angeles. Debris basins and check dams are special types of control structures that are used to trap and store bed-load sediments.

b. Stabilizers.

(1) A stabilizer is generally placed normal to the channel center line and traverses the channel invert. When the stabilizer crest is placed approximately at the elevation of the existing channel invert, it may consist of grouted or ungrouted rock, sheet piling, or a concrete sill. The stabilizer should extend into or up the channel bank and have adequate upstream and downstream bed and bank protection. Plate 44 illustrates the grouted stone type of stabilizer used in USAED, Los Angeles. Stabilizers may result in local flow acceleration accompanied by the development of scour holes upstream and downstream. As indicated in Plate 44, dumped stone should be placed to anticipated scour depths. Maximum scour depths usually occur during peak discharges.

(2) Laboratory tests on sheet piling stabilizers for the Floyd River Control Project were made by the University of Iowa for USAED, Omaha (Linder 1963). These studies involved the development of upstream and downstream bed and bank riprap protection for sheet piling stabilizers in a channel subject to average velocities of 14 fps. The final design resulting from these tests is shown in Plate 45. Plate 46 is a general design chart giving derrick stone size required in critical flow areas as a function of the degree of submergence of the structure. Plate 47 presents design discharge coefficients in terms of the sill submergence T and critical depth d_c for the channel section. Use of Plates 46 and 47 is predicated on the condition that the ratio T/d_c is greater than 0.8. For smaller values the high-velocity jet plunges beneath the water surface, resulting in excessive erosion. The top of the sheet piling is set at an elevation required by the above-mentioned criteria. Plate 47 is used with the known discharge to compute the energy head at $5d_c$ upstream of the structure. The head H on the structure is determined from the energy equation and used with Plate 46 to estimate the required derrick stone size. The curves in Plates 29 and 30 should be used as guides in the selection of riprap sizes for the less critical flow area.

c. Drop structures.

(1) Description and purpose. Drop structures are designed to check channel erosion by controlling the effective gradient, and to provide for abrupt changes in channel gradient by means of a vertical drop. They also provide a satisfactory means for discharging accumulated surface runoff over fills with heights not exceeding about 5 ft and over embankments higher than 5 ft provided the end sill of the drop structure extends beyond the toe of the embankment. The hydraulic design of these structures may be divided into two general phases, design of the notch or weir and design of the overpour basin. Drop structures must be so placed as to cause the channel to become stable. The structure must be designed to preclude flanking.

(2) Design rules. Pertinent features of a typical drop structure are shown in Plate 48. Discharge over the weir should be computed from the equation $Q = CLH^{3/2}$, using a C value of 3.0. The length of the weir should be such as to obtain maximum use of the available channel cross section upstream from the structure. A trial-and-error procedure should be used to balance the weir height and width with the channel cross section. Stilling basin length and end sill height should be determined from the design curves in Plate 48. Riprap probably will be required on the side slopes and on the channel bottom immediately downstream from the structure.

d. Debris basins and check dams.

(1) General. Debris basins and check dams are built in the headwaters of flood control channels having severe upstream erosion problems in order to trap large bed-load debris before it enters main channels. This is done to prevent aggradation of downstream channels and deposition of large quantities of sediment at stream mouths. Also, the passage of large debris loads through reinforced concrete channels can result in costly erosion damage to the channel. Such damage also increases hydraulic roughness and reduces channel capacity. A general summary of data on the equilibrium gradient of the deposition profile above control structures has been presented by

Woolhiser and Lenz (1965). The principles of design and operation of large debris basins as practiced by USAED, Los Angeles, have been presented by Dodge (1948). Ferrell and Barr (1963) discuss the design, operation, and effects of concrete crib check dams used in the Los Angeles County Flood Control District on small streams.

(2) Debris storage. Debris basins, usually located near canyon mouths at the upper end of alluvial fans, are designed to settle out and provide storage space for debris produced from a single major storm. In the Los Angeles area, the debris basin design capacity has been based on 100,000 cubic yards (cu yd) per square mile of drainage area, or 62 acre-feet per square mile. This quantity was obtained as an envelope curve of observed debris production during the storm of 1938 (Dodge 1948). Later estimates by Tatum (1963), taking into account factors affecting debris production such as fire history of the area, indicated a value of about twice this amount. Debris storage in the basin is usually maintained by reexcavation after a major storm period. The debris stored in the basin after any one flood should not be allowed to exceed 25 percent of the basin capacity. When permanent debris storage is more economical than periodic excavation, the average annual rate of debris accumulation multiplied by the project life should be used for storage capacity. Data from the Los Angeles County Flood Control District (Moore, Wood, and Renfro 1960) on 49 debris dams and basins give a mean annual debris production of 5,500 cu yd per square mile of drainage basin. This figure applies in the Los Angeles and similar areas, and can be used to determine the economic feasibility of long-term storage versus periodic debris removal.

(3) Debris basin elements. A debris basin consists of five essential basic parts:

(a) A bowl-shaped pit excavated in the surface of the debris cone.

(b) An embankment, usually U-shaped in plan, constructed from pit material, located along the two sides and the downstream end of the pit, and joining the hillside at each end where possible.

(c) One or more inlet chutes at the upstream end of the pit, when necessary to prevent excessive streambed degradation upstream of the debris basin.

(d) A broad-crested spillway at the downstream end of the basin leading to a flood control channel.

(e) An outlet tower and conduit through the embankment at the spillway for basin draining.

Plate 49 shows general design plans for a debris basin. The basin shape, the inlets, and the outlet should be located so that the debris completely fills the basin before debris discharge occurs over the spillway.

(4) Design criteria. The slope of the upper surface of the debris deposit must be estimated to determine the proper basin shape and to estimate the total debris capacity of the basin. A value of 0.5 times the slope of the natural debris cone at the basin site has been used for design. The basin side embankments should be of sufficient height and extend far enough upstream to confine the maximum debris line slope projected upstream from the spillway crest. The spillway should be designed to pass the design flood discharge with the basin filled with debris. The tops of the basin embankments should provide 5 ft of freeboard with the foregoing conditions. The design criteria for debris basins in the Los Angeles area should be used only for general guidance because of large differences in geology, precipitation patterns, land use, and economic justification in different parts of the country. The following conditions are peculiar to the Los Angeles area:

(a) Phenomenal urban growth in the desirable land area of the lower alluvial fans.

(b) Large fire potential.

(c) Hot, dry climate over a large portion of the year which inhibits vegetative growth.

(d) Sudden torrential rainfall on precipitous mountain slopes during a short rainy season.

(e) Unstable soil conditions subject to voluminous slides when saturated.

Debris and sediment production rates vary throughout the country depending on many factors, some of which are controllable by man. Extensive construction, strip mining operations, intensive agricultural use, and timber cutting operations are only a few examples of land uses that can have a profound local effect on sediment production and thus determine the type of sediment control necessary. Formulation of a sediment control plan and the design of associated engineering works depend to a large extent on local conditions.

4-2. Air Entrainment

a. General. Air entrainment should be considered in the design of rapid- flow channels. The entrainment of air may result in bulking of the flow and necessitate increased wall heights. Presently available data indicate that appreciable air entrainment should not occur with Froude numbers less than about 1.6.

b. Early design criteria. The USAED, Sacramento, developed the following equation based on data reported by Hall (1943):

$$m = \frac{V^2}{200gd} \tag{4-1}$$

where

m = air-water ratio

V = theoretical average flow velocity without air

d = flow depth including air

The term V^2/gd is the Froude number squared. Equation 4-1 with minor differences in the definition of terms has been published by Gumensky (1949). The basic equation has been used extensively for design purposes in the past.

c. Modern investigations. The mechanics of self-aerated flow in open channels with sand grain surfaces has been studied at the University of Minnesota by Straub and Anderson (1960). The results of the Minnesota tests have been combined with selected Kittitas chute prototype data (Hall 1943) and published as HDC 050-3. The chart includes the following suggested design equation:

$$\bar{C} = 0.701 \log_{10} \left(\frac{S}{q^{1/5}} \right) + 0.971 \tag{4-2}$$

where

$\bar{C}$ = ratio of experimentally determined air volume to air plus water volume

S = sine of angle of chute inclination

q = discharge per unit width of channel

d. Design criteria. Use of Equation 4-2 or HDC 050-3 requires the assumption that the experimental water flow depth d_w in the term $\bar{C} = d_a/(d_a + d_w)$ where d_a is depth of air-water mixture, ft, is the same as the theoretically computed flow depth. The Minnesota data indicate that this assumption is valid only for small Froude numbers. For large Froude numbers, the theoretically computed depths for nonaerated flow were found to be 50 to 75 percent greater than the observed experimental flow depth. For this reason and for convenience of design, the Minnesota and Kittitas data have been computed and plotted in terms of the observed total flow depth (air plus water) and the theoretical flow depth and Froude number for nonaerated flow (Plate 50a). The resulting design curve has been extrapolated for low Froude numbers and replotted as Plate 50b. This plate should be used for air-entrained flows in flood control channels. A comparison of HDC 050-3 and Plate 50b indicates that this plate results in more conservative design for low Froude numbers.

4-3. Hydraulic Jump in Open Channels

a. General. Flow changes from the rapid to tranquil state will usually occur in the form of a hydraulic jump. The hydraulic jump consists of an abrupt rise of the water surface in the region of impact between rapid and tranquil flows. Flow depths before and after the jump are less than and greater than critical depth, respectively. The zone of impact of the jump is accompanied by large-scale turbulence, surface waves, and energy dissipation. The hydraulic jump in a channel may occur at locations such as:

(1) The vicinity of a break in grade where the channel slope decreases from steep to mild.

(2) A short distance upstream from channel constrictions such as those caused by bridge piers.

(3) A relatively abrupt converging transition.

(4) A channel junction where rapid flow occurs in a tributary channel and tranquil flow in the main channel.

(5) Long channels where high velocities can no longer be sustained on a mild slope.

b. Jump characteristics.

(1) The momentum equation for the hydraulic jump is derived by setting the hydrodynamic force plus momentum flux at the sections before and after the jump equal, as follows:

$$A_1\bar{y}_1 + \frac{Q^2}{gA_1} = A_2\bar{y}_2 + \frac{Q^2}{gA_2} \qquad (4\text{-}3)$$

where $\bar{y}$ is the depth to the center of gravity of the stream cross section from the water surface. For a rectangular channel the following jump height equation can be obtained from Equation 4-3:

$$\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{1 + 8F_1^2} - 1\right) \qquad (4\text{-}4)$$

where the subscripts 1 and 2 denote sections upstream and downstream of the jump, respectively. Equation 4-3 also gives good agreement for trapezoidal channels as shown by tests reported by Posey and Hsing (1938). However, flood channels should not be designed with jumps in trapezoidal sections because of complex flow patterns and increased jump lengths.

(2) The energy loss in the hydraulic jump can be obtained by use of the energy equation and the derived jump height relation (Chow 1959). This results in an equation that is a function only of the upstream Froude number. The relations between the Froude number, the jump height (Equation 4-4), and the energy loss (Equation 15-1, Brater and King 1976) are presented in Plate 51. The relation between the Froude number and the jump length, based on the data by Bradley and Peterka (1957) for rectangular channels, is also presented in this plate.

c. Jump location.

(1) The location of the hydraulic jump is important in determining channel wall heights and in the design of bridge piers, junctions, or other channel structures, as its location determines whether the flow is tranquil or rapid. The jump will occur in a channel with rapid flow if the initial and sequent depths satisfy Equation 4-3 (Equation 4-4 for rectangular channels). The location of the jump is estimated by the sequent depths and jump length. The mean location is found by making backwater computations from upstream and downstream control points until Equation 4-3 or 4-4 is satisfied. With this mean jump location, a jump length can be obtained from Plate 51 and used for approximating the location of the jump limits. Because of the uncertainties of channel roughness, the jump should be located using practical limits of channel roughness (see paragraph 2-2c). A trial-and-error procedure is illustrated on page 401 of Chow (1959).

(2) The wall height required to confine the jump and the backwater downstream should extend upstream and downstream as determined by the assumed limits of channel roughness. Studies also should be made on the height and location of the jump for discharges less than the design discharge to ensure that adequate wall heights extend over the full ranges of jump height and location.

(3) In channels with relatively steep invert slopes, sequent depths are somewhat larger than for horizontal or mildly sloping channels and jump lengths are somewhat smaller than those given in Plate 51. Peterka (1957) summarizes the available knowledge of this subject. This reference and HDC 124-1 should be used for guidance when a jump will occur on channel slopes of 5 percent or more.

d. Undular jump. Hydraulic jumps with Froude numbers less than 1.7 are characterized as undular jumps (Bakhmeteff and Matzke 1936) (see Plate 52). In addition, undulations will occur near critical depth if small disturbances are present in the channel. Jones (1964) shows that the first wave of the undular jump is considerably higher than given by Equation 4-4. The height of this solitary wave is given by

$$\frac{a}{y_1} = F_1^2 - 1 \qquad (4\text{-}5)$$

where a is the undular wave height above initial depth y_1. Additional measurements were also made by Sandover and Zienkiewicz (1957) verifying Equation 4-5 and giving the length of the first undular wave. Other measurements with a theoretical analysis have been reported by Komura (1960). Fawer (Jaeger 1957) has also given a formula for the wavelength based on experimental data; Lemoine (Jaeger 1957) used small-amplitude wave theory to give the wavelength of the undular jump. The results of these investigations are summarized in Plate 52, which gives the undular jump surge height, breaking surge height (Equation 4-4), and the wavelength of the first undular wave. Also shown in this plate is a relation given by Keulegan and Patterson (1940) for the height of the first undulation

$$\frac{a}{y_1} = \frac{3}{2}\left(\frac{y_2 - y_1}{y_1}\right) \qquad (4\text{-}6)$$

Experiment and theory indicate that the undular wave will begin to spill at the first crest when the Froude number exceeds about 1.28. Undulations persist, however, until the Froude number exceeds about $\sqrt{3}$ ($\approx$ 1.7). This is the limit for breaking waves when Equation 4-4 gives a value of $y_2/y_1 = 2$. Further configuration information on undular jumps may be obtained from Figures 44, 45, and 46 of USBR (1948).

e. Stilling basins. Stilling basin design for high Froude numbers is covered in EM 1110-2-1603. The design of stilling basins in the range of Froude numbers from 1.0 to about 1.3 is complicated by undular waves that are dissipated only by boundary friction with increasing distance downstream. This range of Froude numbers should be avoided whenever possible because of flow instability. The hydraulic jump with Froude numbers of 1.3 to 1.7 is characterized by breaking undulations with very little energy dissipation (see Plate 51). Wall heights in this range of Froude numbers should be designed to contain waves up to the value given by the Keulegan and Patterson (1940) limit.

4-4. Open Channel Junctions

a. General. The design of channel junctions is complicated by many variables such as the angle of intersection, shape and width of the channels, flow rates, and type of flow. Appendix E presents a theoretical analysis, based on the momentum principle, that can be used for several types of open channel junctions. The design of large complex junctions should be verified by model tests.

b. Wave effects.

(1) Standing waves (Ippen 1951) in rapid flow at open channel junctions complicate flow conditions. These waves are similar to those created in channel curves described in paragraph 2-4, and may necessitate increased wall heights in the vicinity of the junction. The studies by Bowers (1950) indicate that a hydraulic jump may form in one or both of the inlet channels, depending on the flow conditions.

(2) Wave conditions that may be produced by rapid flow in and downstream of a typical junction are shown in Plate 53. One area of maximum wave height can occur on the side channel wall opposite the junction point and another on the main channel right wall downstream from the junction. Behlke and Pritchett (1966) have conducted a series of laboratory tests indicating that wave pileup against the channel walls can be up to 7 times the initial depth with a flow Froude number of 4. The design of walls to contain these wave heights over long channel distances is usually not economical. The practical remedy is to reduce or minimize standing waves.

(3) Peak flows from the side channel may not occur simultaneously with peak flows in the main channel. Laboratory tests by Behlke and Pritchett (1966) indicate that occurrence of the design flow in one of the channels with zero flow in the other can result in very high wave pileup on the junction walls. Plates 54a and b show maximum wave height as a function of upstream Froude number for conditions of zero flow in the side channel and main channel, respectively. This plate demonstrates the need for keeping the angle of the junction intersection relatively small. The data are also useful in designing wall heights; for example, the maximum wave pileup on the main channel wall would be greater than twice the side channel flow depth for $F_2 = 3.0$, a junction angle of 15 deg, and no flow in the main channel.

c. Wave height criteria. Behlke and Pritchett's (1966) recommended criteria for the design of channel junctions in rapid flow to minimize wave effects are listed below:

(1) Enlarge the main channel below the junction apex to maintain approximately constant flow depths throughout the junction.

(2) Provide equal water-surface elevations in the side and main channels in the vicinity of the junction.

(3) Ensure that the side channel wave originating at the junction apex impinges on the opposite side channel wall at its intersection with the enlarged main channel wall.

(4) Provide tapered training walls between the main channel and the side channel flows.

(5) Ensure that maximum wave heights occur with maximum flows. Plate 55 illustrates typical design examples for rectangular and trapezoidal channels using these criteria. Important junctions in rapid flow designed to reduce wave effects should be model tested at all probable flow combinations as well as at design flow.

d. Confluence design criteria.

(1) The results of several model studies in USAED, Los Angeles, indicate that some general guides can be adopted for the design of confluence junctions. Gildea and Wong (1967) have summarized some of these criteria:

(a) The design water-surface elevations in the two joining channels should be approximately equal at the upstream end of the confluence.

(b) The angle of junction intersection should be preferably zero but not greater than 12 deg.

(c) Favorable flow conditions can be achieved with proper expansion in width of the main channel below the junction.

(d) Rapid flow depths should not exceed 90 percent of the critical depth (Froude number should be greater than 1.13) to maintain stable rapid flow through the junction (paragraph 2-2d(1)).

(2) Model tests of many confluence structures indicate very little crosswave formation and turbulence at the junction if these criteria are followed. Moreover, experience has shown that the momentum equation approach given in Appendix E can be used for junctions involving small angles and equal upstream water-surface elevations.

(3) Typical confluence layouts model tested by USAED, Los Angeles, and proven to have good flow characteristics are shown in Plate 56. The design with the offset in the main channel center line is normally used (Plate 56a). When the main channel center-line alignment cannot be offset, a layout with a transition on the wall opposite the inlet side should be used (Plate 56b). The proper amount of expansion in the main channel downstream of the confluence is very important in maintaining good flow conditions. Plate 57 gives the USAED, Los Angeles, empirical curve for the required increase in channel width, Δb_3, as a function of the discharge ratio. If the junction angle is zero, the width of the channel at the confluence will be equal to the sum of the widths of the main and side channels plus the thickness of the dividing wall between the channels. If a reduction in width is required downstream from the confluence, the transition should be made gradually.

e. Design procedure. The design procedure for the typical open channel confluence shown in Plate 56 involves the following steps:

(1) Determine side-channel requirements relative to discharge, alignment, and channel size.

(2) Select junction point to obtain an entrance angle less than 12 deg. This angle requirement may necessitate a long, spiral curve for the side channel upstream from the junction.

(3) Determine the increase of channel width Δb_3 from the Q_2/Q_3 ratio curve in Plate 57. Compute the required downstream channel width $b_3 = b_1 + \Delta b_3$ and the confluence width $b_c = b_1 + 2\Delta b_3$.

(4) Make the confluence layout on a straight-line basis by setting the main channel walls parallel to and at distances of $(1/2)b_3$ and $b_c - (1/2)b_3$ from the center line as shown in Plate 56a.

(5) Connect the left walls of the side and the main channels by a curve determined by the apex angle θ and a radius r_L given by

$$r_L = \frac{4V^2 b_2}{gy} + 400 \tag{4-7}$$

Equation 4-7 results from a study of a number of confluences built by USAED, Los Angeles. The term $(4V^2b_2)/gy$ is the same as that used in Equation 2-34.

(6) Make the right wall of the side channel concentric with the left wall and locate the junction intersection point. The right wall radius r_R is given by

$$r_R = r_L + b_2 \tag{4-8}$$

(7) Determine the average depth of flow at midpoint of the confluence by the momentum method (Appendix E) assuming $b_m = (1/2)(b_1 + b_2 + b_c)$.

(8) Set the side-channel invert elevation so that the design water-surface levels in both channels approximate

each other. A stepped invert in either of the channels may be required.

(9) Determine the length of transition and invert slope required to reduce the channel width from b_c to b_3 without exceeding the criterion $y/y_c \leq 0.90$ in the transition. Convergence rates should be in agreement with those recommended in paragraph 2-4.

f. Side drainage inlets. Flow disturbances occur where storm drains or industrial waste lines discharge into flood control channels, commonly referred to as "inlets." Small side-drainage flows are commonly conveyed in a pipe storm drain system. Criteria for box and pipe culvert inlet design are given in h below. Economical design for intermediate tributary flows normally requires free surface structures. A side-channel spillway type of inlet for this range of discharge has been developed by USAED, Los Angeles, which reduces disturbances to a minimum in the main channel. This type of junction is described in g below. The conventional confluence structure described in d above should be used for large tributary discharges.

g. Side-channel drainage inlet.

(1) The side-channel spillway type of drainage inlet was developed and model tested by USAED, Los Angeles (1960b). The recommended structure consists of a common wall between the side channel and the main channel. A weir notched in this wall allows the tributary flow to enter the main channel with minimum disturbance. A typical design of this type of structure is illustrated in Plate 58. A small drain should be placed at the lowest point of the side channel. The objective of this design is to discharge the side flow with reduced velocity into the main channel gradually over a relatively long spillway inlet. Model tests (USAED, Los Angeles, 1960b) indicate that this effectively reduces wave action and disturbances in the main channel for all flow combinations. Satisfactory operation may require periodic sediment removal from behind the weir.

(2) The procedure for designing the side-channel spillway inlet structure follows:

(a) Set the spillway crest 0.5 ft above the parallel to the design watersurface level in the main channel.

(b) Determine the required length L of the crest by the equation, $L = Q/(CH^{3/2})$, so that the maximum H is not greater than 1.5 ft with critical depth over the crest C equal to 3.097.

(c) Determine the side-channel flow depth d at the upstream end of the spillway.

(d) Set the side-channel invert so that the spillway approach depth is equal to $d - H$.

(e) Determine the side-channel convergence required to maintain a constant flow depth in the side channel behind the spillway. This should result in a reasonably constant unit discharge over the spillway equal to that computed by the equation in (b) above.

(f) Plot the computed side-channel alignment points obtained from step (e) on the channel plan and connect them by a smooth curve or straight line to intersect the main channel wall so that the side channel has a minimum width of 2 ft behind the spillway.

(g) Adjust the side-channel convergence and repeat step (e) if the spillway length in step (f) does not approximate that determined in step (b).

h. Box and pipe culvert inlets. Gildea and Wong (1967) have determined design criteria for pipe inlets. The variables to be considered in the design are width of the main channel, angle of entrance of the storm drain, size of the storm drain, volume and velocity of flow, and elevation of the storm drain with respect to the channel bottom. Model tests (USAED, Los Angeles, 1960b, 1964) have shown that flow disturbances in the main channel are minimized when side-drain openings are small and side- drainage flows are introduced reasonably parallel to the main flow. The following criteria should be used for design:

(1) The maximum angle of entrance for side culverts should be:

(a) 90 deg for diameters of 24 in. or less.

(b) 45 deg for diameters from 24 to 60 in.

(c) 30 deg for diameter 60 in. or greater.

(2) The culvert invert should be placed no more than 18 in. above the main channel invert to give the maximum submergence practicable.

(3) Automatic floodgates or flap gates should be installed when damage from backflooding from the main channel would exceed that resulting from local pondage caused by gate operation. These gates should be recessed

to prevent projecting into the main channel flow when in a full-open position. Head loss coefficients for flap gates are given in HDC 340-1.

4-5. Hydraulic Model Studies

a. General. The use of hydraulic models has become a standard procedure in the design of complex open channels not subject to analytical analyses or for which existing design criteria based on available model and field tests are inadequate. Hydraulic models afford a means of checking performance and devising modifications to obtain the best possible design at minimum cost. Model tests should be used to supplement but not replace theoretical knowledge, good judgment, and experience of the design engineer. They often indicate design changes that save substantial amounts in construction costs as well as effect improvements in operation. Model tests of large flood control channels are generally desirable where supercritical flow results in standing waves and other major disturbances in channels containing junctions, transition structures, alignment curvature, multiple bridge piers, or stilling basins.

b. Model design.

(1) The theory of model design is treated in EM 1110-2-1602 and other publications (Rouse 1950, Davis and Sorenson (1969), American Society of Civil Engineers (ASCE) 1942). For open channel models, the gravity force will dominate the flow and similitude will require equality of Froude number in the model and prototype. The Froudian scale relations (model-to-prototype) in Table 4-1 apply to undistorted models. The length ratio L_r is the model-to- prototype ratio L_m/L_p. These transfer relations are based on equal force of gravity and density of fluid in model and prototype. The procedure for initiation of model studies is discussed in EM 1110-2-1602.

(2) Model scale ratios for flood control channels have ranged from 1:15 to 1:70, depending on the type of problem being studied, the relative roughness of the model and prototype, and the size of the prototype structure. Scale ratios of 1:15 to 1:30 are usually employed where supercritical flow wave problems are involved. They are also used for sectional models of drop structures, spillways, etc. The smaller scale ratios (1:30 to 1:70) are used for general model studies where long channel lengths are reproduced. The accuracy of possible model construction and flow measurements may control the permissible scale ratios. Most models of channels are generally built to give depths of flow about 0.5 ft or more and channel widths of about 1 to 2 ft. The most common scale ratios used by the USAED, Los Angeles, Hydraulic Laboratory for channel model studies are from 1:25 to 1:40.

c. Model roughness. Turbulent flow will prevail with model channel velocities and depths commonly used in testing. In most cases, the channel flow is rough-turbulent or nearly so; therefore, hydraulic resistance is determined primarily by the relative size of the roughness elements. However, the model Reynolds number will always be smaller than the prototype, and this will to some extent cause scale distortion of certain phenomena such as zones of separation, wave dissipation, flow instability, and turbulence in the model. Particular care should be taken in interpreting those effects that are known to be strongly dependent on viscous forces.

d. Slope distortion. An empirical equation of the Manning type may be used to give the required model roughness (Rouse 1950) for large-scale models where fully rough-turbulent flow prevails. This condition is expressed by the equation

$$n_r = L_r^{1/6} \tag{4-9}$$

If this roughness criterion cannot be fulfilled, slope adjustment or distortion must be applied to the model so that prototype flow conditions can be simulated in the model. The amount of additional slope required is given by the equation (Rouse 1950)

Table 4-1
Scale Relations

Length	Area	Volume	Time	Velocity	Discharge	Manning's n
L_r	L_r^2	L_r^3	$L_r^{1/2}$	$L_r^{1/2}$	$L_r^{5/2}$	$L_r^{1/6}$

$$S_r = \frac{n_r^2}{L_r^{1/3}} \tag{4-10}$$

Equation 4-10 applies only when the model and prototype channels are geometrically similar in cross section. Without slope distortion ($S_r = 1$), this equation would reduce to Equation 4-9.

e. Scale distortion.

(1) Distorted scales are generally used in models of river channels, floodways, harbors, and estuaries. Movable-bed models are distorted in order to ensure the movement of particle-size bed material under model flow conditions. Flood control projects for the improvement of river channels through urbanized areas often require the reproduction of long channel lengths and wide floodway widths. Most such channels have mild slopes and the flows are tranquil at very low Froude numbers. In order to fit this type of model in a reasonably economical space, the horizontal scale ratio has to be limited and vertical scale distortion selected to give measurable depths and slopes as well as to ensure turbulent flow in the model. The use of distorted models should be generally limited to problems involving tranquil flows. A number of reports (USAEWES 1949a, 1949b, 1953) have been published that illustrate the application of distorted models for the solution of complex local flood protection problems and channel improvements.

(2) The scale relations for distorted models are given in ASCE (1942). If the bed slope ratio is made equal to the energy slope ratio, the slope ratio will also be equal to the amount of model distortion.

$$S_r = \frac{y_r}{L_r} \tag{4-11}$$

where y_r is the vertical scale ratio and L_r is the horizontal scale ratio, model to prototype. The Manning equation can then be used to obtain a roughness criteria for model design (Rouse 1950).

$$n_r = \frac{R_r^{2/3}}{L_r^{1/2}} \tag{4-12}$$

For a wide channel Equation 4-12 reduces to

$$n_r = \frac{y_r^{2/3}}{L_r^{1/2}} \tag{4-13}$$

The required roughness in the model can be computed by Equation 4-12 and used as a guide in designing the model. Distorted models should be verified using measured field data or computed prototype data prior to testing of improvement plans. Flood control channel models should be built to as small a distortion as is economically feasible. A distortion of 3 or less is desirable, but depends to some extent on the type of information needed from the model study. It may sometimes be economically feasible to divide a long channel study into several problem areas and model each one independently. In this manner different scales could be used as required by the problem to be studied in each reach.

f. Movable-bed models. Open channel studies involving problems of sediment erosion, transportation, or deposition require a bed of sand or other material that will move when subjected to flow. Rouse (1950), Davis and Sorenson (1969), and ASCE (1942) give considerable detail on design, construction, verification, and use of movable-bed models. Qualitative indication of bed movement has been used in flood control channel models for design purposes. For example, the effectiveness of a hydraulic jump to dissipate energy is often obtained through the relative extent of downstream scour. The stability of riprap protection can also be obtained from model studies. A typical example of a study to determine the relative scour and design of riprap protection at inlet and outlet channels is given in USAED, Los Angeles (1960a).